Some Studies on Engineering Properties, Problems, Stabilization and Ground Improvement of Lithomargic Clays

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ABSTRACT: The study area for this paper is coastal Karnataka in India. The area has laterites and lateritic soils, and also a large number of sporadic lateritic hillocks. The soil stratification mainly consists of lithomargic clay sandwiched between the weathered laterite at top and the hard granitic gneiss underneath. Quite often the top laterites are removed in this area for use as bricks for construction purposes, thus exposing the underlying lithomargic clay. This coastal area receives copious amount of rainfall and a lot of developmental activities are taking place. These lithomargic clays, locally called as 'shedi soils' are also used as fill material in low lying areas, very often adjacent to water bodies. These soils behave as dispersive soils and are also highly erosive. A lot of engineering problems - such as foundation problems, subgrade problems, erosion and slope stability problems are being faced due to the presence of these shedi soils. Some laboratory studies on the engineering and strength properties of these lithomargic clays and stabilized soils, Ground Improvement on shedi grounds are made and reported.

KEYWORDS: Lithomargic clays, Dispersive soil, Erosion, Slope stability, Stabilization, Geogrid reinforcement, Stone columns, Coir fibres, Geocoir, Vetiver

1. INTRODUCTION

1.1 Study Area

The study area for this paper is coastal Karnataka in western peninsular India comprising of three districts namely Uttara Kannada (UK), Udupi and Dakshina Kannada (DK). A lot of developmental activities are taking place in this area. This area has quite a few rivers that flow westwards and exit into the Arabian Sea. The area has laterites and lateritic soils, and also a large number of sporadic lateritic hillocks. The top laterites are used as bricks for construction purposes in this area. Lithomargic clay is a product of laterization and underlies the top hard and porous lateritic crust. The lithomargic clay behaves like a dispersive soil. The lithomargic clay is present between the weathered laterite at top and the hard granitic gneiss underneath. Lithomargic clays (shedi soils) are also used for construction purposes, for backfilling purposes in low lying areas. A lot of engineering problems are being faced due to the presence of this shedi soil, either naturally or due to backfilling, and the fluctuating water levels, some of which are being discussed in this paper. This study focuses on the properties of lithomargic clays of coastal Karnataka. It also discusses some methods of soil stabilization and ground improvement on shedi grounds from laboratory studies.

This study is focused more on DK district areas. Mangalore is the administrative headquarters of the DK district and is a major port city of India. Being a coastal city, Mangalore has also a great potential for rapid growth and industrialization. Many large-scale industries have come up and are providing jobs to many. A large petroleum refinery, chemical and fertilizer factory, iron ore company etc. have already been established and are functioning. Many more mega projects are due to come in near future, especially power projects. Most of these require lots of infrastructural facilities and some of these may have be located on poor subsoil conditions.

This coastal area, which is adjacent to the Arabian Sea, receives copious amount of rainfall. Average annual rainfall of the area is about 3500 mm to 4000 mm. Climate in this area, in addition to the heavy rainfall, is marked by high humidity and little changes in temperatures. March, April and May are the hottest months of the year and temperatures are around 33 to 38 degrees centigrade. During monsoon season (June to September), the temperatures are around 30 degrees centigrade. Humidity is high throughout the year. It is about 85% during southwest monsoon due to heavy precipitation and about 65% in the month of February.

1.2 Soil Stratification

The soil stratification in this coastal Karnataka area, consists of hard (vesicular layer) and highly porous laterite at the surface (1-3 m thick) underlain by the lithomargic clay (up to about 8 m thick). Lithomargic clays are products of tropical weathering often referred to as laterization. Laterization is a chemical weathering process, of the parent rock, which is granitic gneiss in this area, due to intense tropical weather conditions i.e. high temperatures and rainfall. The clay minerals present in the parent rock i.e. granites and granitic gneisses are broken down, where upon their silica are released and removed by leaching. The residual lateritic soil consists largely of aluminium oxide or of hydrous iron oxides. This lithomargic clay is locally called as the 'shedi soil'. Shedi soils of coastal Karnataka generally classify, based on grain size distribution, as silty sands or sandy silts, with very little clay size particles. Even the behaviour of the fine fraction (75 micron down soils) is nowhere near to the clay behaviour. The soil (LL and PI point) falls below the A-line and the soil has very little or no cohesion. Its behaviour is more of a silty or of non-plastic nature. So the term 'clay' in 'lithomargic clay' is a misnomer. Shedi soils, are available in varying colours such as whitish, yellowish, pinkish etc.

1.3 Laterites and Lithomargic Clays

Buchanan (1807) [Ref: wikipedia] was the first to coin the term laterite, to describe ferruginous, vesicular, unstratified and porous soil with yellow ochres due to high iron content in Malabar, India. Later in Latin means brick. Laterites are found in many places around the world such as Africa, South America, Arabian peninsula and Australia. Laterites are soil types rich in iron and aluminium formed in hot and humid tropical areas. Nearly all laterites are rusty-red because of iron oxides.

There are outcrops of laterite throughout the Konkan area that extends along the western coast of India from Cochin to Mumbai, including the whole of coastal Karnataka. Underlying the top gravelly laterites are lithomargic clays. Formation of laterites, properties of laterites and lithomargic clays has also been extensively studied and reported by many researchers, to name a few, Roy Chowdhury et al. (1965) Gidigasu (1972,1976), Morin and Todor (1975) and others. Lateritic soils have an amorphous blend of Al2Si2O5(OH)4. Soils with 50-90% lateritic constituents, and less of the lithomargic constituents, are known as lithomargic laterites. Soils with 25-50% laterite content, and more of the lithomargic constituents are known as lateritic lithomarges. The soils being analyzed in this paper are the lateritic lithomarges. Lateritic lithomarges, with a higher percentage (75 to 100%) of lithomargic clay (which are products of laterization), are more often used for filling purposes in low lying areas.

2. PROPERTIES OF LITHOMARGIC CLAYS

Shedi soil samples (Sample Nos. 1 to 10) were collected from ten sites along national highway NH 17 of D.K. district (Rao, 2008), and another 15 samples (Sample Nos. 11 to 25) were collected from in and around the NITK campus (Table 1) [Shivashankar et al. 2014a]. The sites that were located by the side of the National Highway were within the limits of the right of way. All the soil samples were subjected to various tests such as the index tests i.e. specific gravity, grain size distribution (sieve analysis and sedimentation analysis (hydrometer analysis)), Atterberg's limits (liquid limit, plastic limit and shrinkage limit); compaction tests; strength tests such as the CBR test under soaked and unsoaked conditions, unconfined compression or uniaxial compression strength tests, direct shear tests, unconsolidated undrained (UU) triaxial shear tests without pore water pressure measurements, laboratory-scale plate bearing tests, fatigue life tests, laboratory erosion tests etc.; and tests for determining chemical composition of soils.

Table 1 Results of Tests for Specific Gravity and Atterberg's Limits on Lithomargic Clay Samples

Sample No.	Sp.Gr.	LL (%)	PL (%)	PI (%)	SL (%)
1	2.59	34.5	21.2	13.3	15.8
2	2.53	31.3	19.9	11.4	11.0
3	2.62	27.2	17.8	9.4	15.6
4	2.53	42.0	24.2	17.8	22.4
5	2.64	41.7	23.4	18.3	21.7
6	2.62	48.0	30.5	17.5	29.7
7	2.72	60.8	31.6	29.2	25.8
8	2.79	42.4	31.1	11.3	24.4
9	2.50	40.5	26.1	14.4	21.0
10	2.54	40.6	23.5	17.1	19.6
11	2.50	38.0	23.6	14.4	20.7
12	2.51	57.0	26.6	30.4	19.4
13	2.70	43.0	27.9	15.1	19.3
14	2.50	39.4	24.0	15.4	17.6
15	2.58	36.8	20.4	16.4	16.5
16	2.60	47.0	34.0	13.0	27.5
17	2.32	37.4	32.9	4.5	25.7
18	2.49	60.6	37.3	23.3	27.0
19	2.61	44.0	41.0	3.0	36.0
20	2.58	62.0	30.4	31.6	21.0
21	2.64	46.5	37.6	8.9	25.7
22	2.55	33.0	22.4	10.6	18.9
23	2.49	44.0	33.3	10.7	30.2
24	2.52	53.0	33.0	20.0	30.5
25	2.60	47.0	37.0	10.0	20.0

2.1 Laboratory Tests and Results

2.1.1 Results of Specific Gravity and Atterberg's Limits Tests

The laboratory tests on the 25 shedi soil samples consisted of determination of specific gravity and Atterberg's limits (Table 1). All the tests are conducted as per the relevant Indian Standard codes [SP36 (Part 1):1987]. Atterberg's tests are performed on fine fraction of soil passing 425 micron sieve.

2.1.2 Results of Particle Size Distribution Tests and Soil Classification

The tests for particle size distribution is done using the sieve analysis method (for soil fractions above 75 microns size), and the hydrometer method (for soil fractions of size lesser than 75 microns). Particle size distribution and behavioral classification as per unified soil classification system (i.e. Indian Soil classification system) are tabulated in Table 2 [SP36 (Part 1):1987].

Table 2 Grain Size Distribution and Classification of Lithomargic Clay Samples

Sample No.	Gravel Size (%)	Sand Size (%)	Silt Size (%)	Clay Size (%)	Classi-fication
1	22.2	43.4	29.1	5.3	SM
2	8.8	70.8	16.3	4.1	SM
3	25.4	55.7	17.5	1.3	SM
4	24.7	46.5	28.8	0.0	SM
5	15.5	29.6	48.8	6.1	MI
6	26.3	38.1	22.6	13.0	SM
7	21.6	47.6	27.8	3.0	SM
8	21.6	63.0	15.4	0.0	SM
9	19.3	51.5	27.2	2.0	SM
10	8.4	37.5	47.7	6.4	MI
11	20.1	52.5	27.4	0.0	SM
12	36.1	48.9	14.0	1.0	SM
13	26.4	49.2	21.6	2.8	SM
14	15.2	46.1	33.7	5.0	SM
15	5.1	76.0	18.1	0.8	SM
16	0.0	30.0	64.0	6.0	MI
17	6.0	94.0	0.0	0.0	SP
18	0.0	34.0	38.0	28.0	MH
19	0.0	50.0	48.0	2.0	SM-MI
20	0.0	26.0	53.0	21.0	CH
21	1.0	56.0	37.0	6.0	SM
22	0.6	42.4	55.0	2.0	ML-MI
23	1.0	27.0	28.0	44.0	MI
24	4.7	21.3	42.1	31.9	MI
25	1.0	45.0	41.0	13.0	SM-MH

2.1.3 Results of Compaction Tests and C.B.R. Tests

Standard Proctor (light) compaction and in a few cases modified Proctor (heavy) compaction tests are performed on shedi soils [SP36 (Part 1):1987]. California Bearing Ratio (CBR) tests are also conducted on soils (1 - 15) at corresponding field densities and field moisture contents as per SP36. In case of Sample No.20, CBR test is conducted at maximum dry density (MDD) and optimum moisture content (OMC) from heavy compaction test.

In case of Sample Nos.21 and 25, MDD and OMC are from light compaction test. CBR tests for sample Nos. 21 and 25 are also conducted corresponding to light compaction. In case of sample No.24, both light and heavy compaction tests are conducted, and CBR tests are conducted corresponding to both light and heavy compactions. The results are tabulated in Table 3. Both unsoaked and soaked (after four days of soaking) CBR tests are conducted. Soaked CBR tests are critical in the design of flexible pavements (all weather roads) under severe climatic conditions, in areas such as DK district which experiences heavy rainfall during rainy seasons and long periods of soaking. The thickness of the upper layers of a flexible pavement depends on the CBR value of the underlying soil subgrade.

Table 3 Results of Field Density, Compaction and California Bearing Ratio (CBR) Tests on Lithomargic Clay Samples

Sample No.	Field Dry Density (kN/m^3)	MDD (kN/m^3)	OMC (%)	CBR_u (%)	CBR_s (%)
1	15.1	18.0	17.6	5.0	3.0
2	15.6	20.7	12.5	3.0	1.0
3	19.7	21.9	10.5	25.0	9.0
4	18.1	20.5	14.5	11.0	4.0
5	17.4	18.7	12.5	20.0	5.0
6	17.8	17.9	18.7	15.0	5.0
7	17.2	19.7	15.7	13.0	5.0
8	17.4	19.9	17.2	17.0	5.0
9	16.1	19.3	17.8	7.0	2.0
10	15.7	17.4	18.9	15.0	4.0
11	16.7	20.3	14.2	16.0	5.0
12	15.3	20.9	12.1	17.0	5.0
13	16.5	20.0	16.0	15.0	4.0
14	16.6	20.3	13.0	5.0	3.0
15	15.9	21.3	10.1	6.0	2.0
16	NA	14.8	25.8	NA	NA
17	NA	14.3	24.5	NA	NA
18	NA	14.2	27.0	NA	NA
19	NA	15.8	20.0	NA	NA
20	NA	15.0 (17.2*)	29.0 (19.0*)	42.0*	2.0*
21	NA	16.2	16.0	18.0	4.0
22	NA	17.3	11.6	NA	NA
23	NA	15.4	21.6	NA	NA
24	19.9	16.5 (18.7*)	15.6 (13.3*)	12.0 (16.0*)	3.0 (4.0*)
25	NA	16.2	20.0	18.0	4.0

- In above table CBR_u is CBR unsoaked and CBR_s is CBR soaked.
- CBR values for Sample Nos. 1 to 15 are at corresponding field densities and field moisture contents
- NA = not available

* values for heavy compaction (Modified Proctor Compaction) tests

2.1.4 Results of Unconfined Compression (U.C.C.) Strength Tests and U.U. Triaxial Compression Tests

Results of unconfined compression tests and UU triaxial shear tests [SP36 (Part 1):1987] are tabulated in Table 4. Unconfined compression and UU triaxial shear tests are conducted on Sample Nos. 1 to 15 on remoulded samples of 38mm diameter and 76 mm height of respective sites compacted to field density and field moisture content. In case of Sample Nos. 17, 21 and 24 unconfined compression tests are conducted at their respective MDD and OMC from standard Proctor compaction tests. Khanna and Justo (1991) report that the tangent modulus obtained at a confining pressure of 0.14 MPa can be used to estimate the modulii of elasticity of the pavement materials. Therefore UU triaxial shear tests are performed on three test specimens of each sample, each at cell pressures of 1 kg/cm^2, 1.4 kg/cm^2 and 2 kg/cm^2 (i.e. at cell pressures of 0.1 MPa, 0.14 MPa and 0.2 MPa) to obtain the shear strength parameters of the soil samples. Soil sample 3 in Table 3 above is found to have a high field density of 19.7 kN/m^3 and it gave a high modulus of resilience of 154 MPa measured using a portable falling weight deflectometer (PFWD) at site. Similarly soil samples 1 and 2 are having low modulii of resilience of about 28 MPa due to their low field densities (Rao, 2008).

Table 4 Results of the Unconfined Compression Strength Tests and UU Triaxial Shear Tests on Lithomargic Clay Samples

Sample No.	q_u (MPa)	Tangent Modulus E_t (MPa)	C (kPa)	ϕ (degrees)	Tangent Modulus $E_{tri\text{-}UU}$ (MPa)
1	0.034	1.81	1.9	26.0	6.21
2	0.056	4.73	5.8	9.0	8.72
3	0.570	41.36	3.1	31.0	70.94
4	0.082	5.24	0.1	41.0	23.28
5	0.273	12.38	10.0	33.0	34.71
6	0.312	20.49	12.0	32.0	27.85
7	0.217	16.04	3.5	22.0	15.10
8	0.237	9.81	8.3	29.0	21.67
9	0.239	12.49	9.2	18.0	16.19
10	0.306	16.31	18.0	12.0	15.16
11	0.309	18.58	0.0	15.0	28.60
12	0.499	21.16	8.7	20.0	22.70
13	0.080	2.67	11.0	8.0	15.08
14	0.069	2.64	0.6	29.0	8.77
15	0.046	4.47	0.5	28.0	20.11
17	0.018	NA	10.0	12.0	NA
17#	0.013	NA	5.5	6.0	NA
19	NA	NA	45.0	25.2	NA
20	0.860*	NA	NA	NA	NA
21	0.087	NA	30.0	21.0	NA
24	0.27 (0.38*)	NA	NA	NA	NA
25	NA	NA	80.0	19.0	10.58$

* at MDD and OMC corresponding to the heavy compaction test.
\# at moisture content of 40% corresponding to the submerged condition in laboratory scale plate bearing bearing tests
$ From plate load test under unsoaked condition. On soaking the value reduces to 3.18

2.1.5 Direct Shear Tests Results on Shedi Soil at Different Moisture Contents

Direct shear tests were conducted on shedi soil [SP36 (Part 1):1987] at different moisture contents on sample No.21. Figures 1 and 2, respectively, show the variation of the shear strength parameters cohesion (C) and angle of internal friction (ϕ) with moisture content. Therein it can be seen that both the cohesion and the angle of internal friction increase rapidly at first with increasing moisture contents. It peaks at around OMC, which is 16%, notably on the dry side of optimum, and then starts to decrease. The decrease is rather rapid in the case of cohesion, while it is somewhat gradual in case of angle of internal friction (Krishna Murthy, 1999). This behavior is a good indication that whenever shedi soils are used in lowland environments in landfills adjacent to water bodies, it should be well compacted and it is very important to control drainage in the post-construction phase.

2.1.6 Laboratory-Scale Plate Bearing Test Results on Shedi Soil at Different Moisture Contents and at Submergence

Results of laboratory scale plate bearing tests on two soil samples, namely Sample No.21 and Sample No.17 are reported herein. In the case of Sample No.21 (Krishna Murthy, 1999), plate loading tests were conducted at moisture contents of 10%, 14%, 16%, 18% and 22% (Figure 3).

All the curves show an increasing trend in load with normalized settlement (s/B, where s is the settlement and B is the size of the footing). The increase is somewhat rapid at first and then mellows down a bit. There is no well defined peak in any of them even after undergoing large settlements. It is also clearly observed that as the moisture content increases beyond OMC, load carrying capacity decreases.

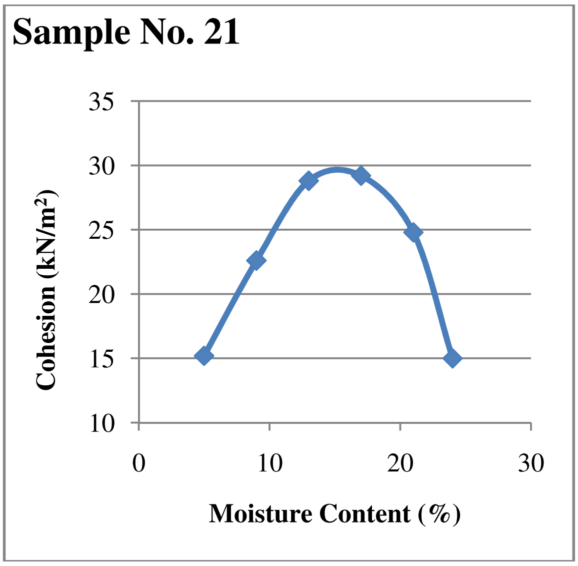

Figure 1 Variation of cohesion of shedi soil with moisture content in case of sample No.21

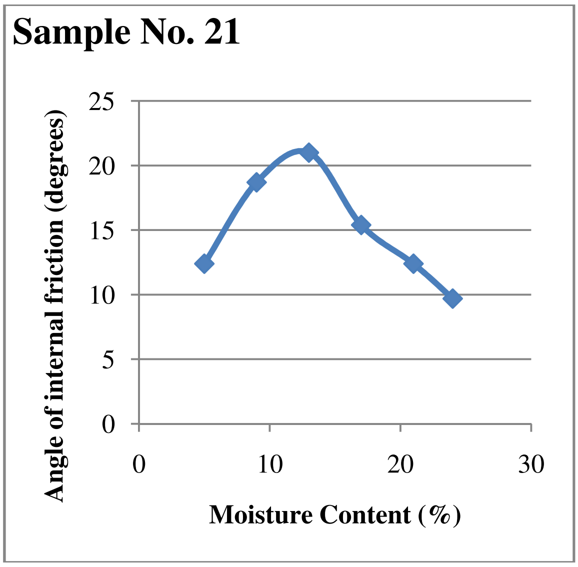

Figure 2 Variation of angle of internal friction of shedi soil with moisture content in case of sample No.21

In the case of Sample No.17, plate loading tests were conducted at OMC of 24.5% (MDD 14.3 kN/m^3) and at moisture content of 40% at a density of 14.3 kN/m^3 (for submerged conditions). Load tests were conducted in a combined test bed and loading frame assembly. The test beds were prepared in a ferrocement tank which is designed keeping in mind the size of the model footing to be tested and the zone of influence. The dimensions of the tank were 750 mm length x 750 mm width x 750 mm depth. The model footing is a rigid mild steel plate of 100 mm x 100 mm size and 20 mm thickness. Further details of the tests are reported in Jayamohan and Shivashankar (2012) and Jayamohan (2014) (Figure 4).

It is seen from Figure 4 that on submergence, the shedi ground loses much of its load bearing capacity. This is due to loss of shear strength of the soil, reduction in cohesion and angle of internal friction, as observed from the results of the laboratory shear strength tests.

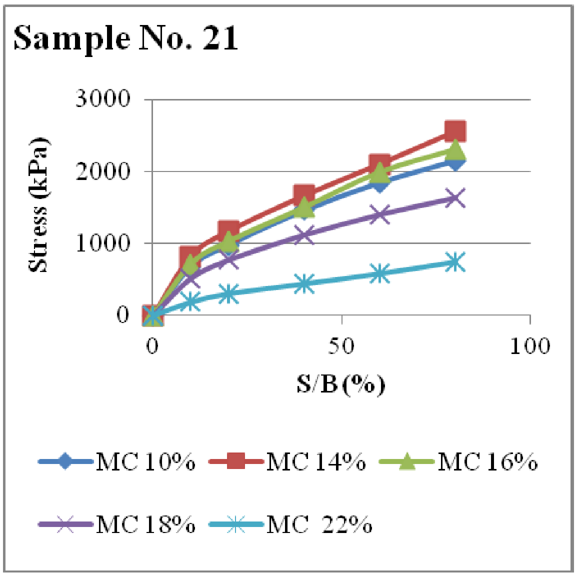

Figure 3 Results of laboratory scale plate bearing tests on shedi soil sample No.21 at different moisture contents

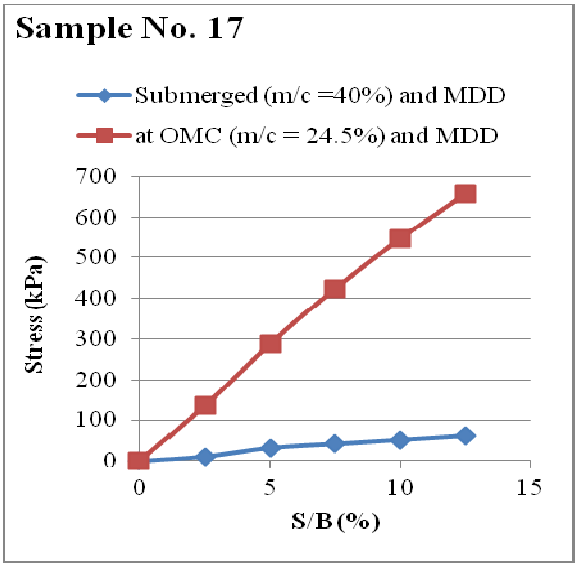

Figure 4 Effect of submergence on shedi soil sample No.17

3. ENGINEERING PROBLEMS DUE TO THE PRESENCE OF SHEDI SOILS

The term soft ground includes soft clay soil, soils with large fraction of fine particles such as silts, clay soil with high moisture content, peat foundations, and loose sand deposits near or under the water table. Shedi soil grounds with a high water content or a high water table easily qualifies as a soft ground and shedi soils in foundation soils and slopes poses a large number of engineering problems. Shedi soils are problematic soils, because they behave like dispersive soils, losing much of their shear strength on wetting and/or due to removal of confinement. They are also highly erosive. It is imperative to solve geotechnical problems concerned with soft, compressible and weak soils such as shedi soils.

3.1 Problems of Low Bearing Capacity, Low Subgrade Strength and Large Settlements

Infrastructural development activities due to rapid urbanization and industrialization, in DK district are forcing the civil engineers to put to best use of even the poorest sites available. Low lying agricultural and marshy lands in and around the Mangalore metropolis are being fast converted into estates by filling with the locally silty soil (shedi soil). These soils are merely dumped and usually not properly compacted. The other scenario is that, very often in areas where the lateritic crust is intact, existing mounds are levelled and the top laterite layer is removed for use as bricks in building constructions, thereby exposing the underlying lithomargic clays. In either case, the infrastructure constructions are invariably to be supported on the poor shedi soil. The problems of such lowlands (shedi grounds) adjacent to water bodies are low bearing capacities and settlements. Further, the sensitivity of these shedi soils to moisture variations, from point of view of their strength, is also of grave concern for highway engineers and foundation engineers, unless these soils are suitably stabilized.

In many projects coming up in the special economic zone (SEZ) of Mangalore and in the refinery area, granular piles are suggested to improve the foundation soil. Figure 5 shows embankment failure on poor foundation soil in lowland environment as well as due to poor compaction. After this experience, the entire project was redone with improved ground using granular piles and proper compaction. For other important and major constructions, such as multistoried constructions, pile foundations are also recommended (Figure 6).

Figure 5 Embankment failure due to poor compaction on untreated soft foundation in lowland environment

Figure 6 Piled foundation work in progress at a shedi soil filled ground/site

On some of the railways tracks, on shedi grounds in excavation situations, along the Konkan Railway (such as the one shown in Figure 7), whenever trains passed by, there would be a spring or jet of soil water from below the trains, hitting the bottom of the trains. The high speed trains running with normal speeds of more than a hundred kilometers per hour were made to crawl on this stretch (at just 10 to 20 km per hour). This mud-pumping was prevented after providing suitable ground improvement (soil stabilization) and supporting the tracks on that stretch over piles (fortunately rock was available at shallow depths of 5 to 6 m).

Figure 7 Failures of a deep excavated shedi soil slope (as seen in the far end) and mud pumping along the Konkan railway track.

3.2 Problems of Slope Stability and Erosion

Mangalore and surrounding places are generally on a sloping terrain. The large number of geotechnical failures that one observes in this area are slope failures. First of this kind are failures of natural slopes like landslides or land slips etc. due to changed drainage patterns because of numerous human activities in the name of development. Secondly, other manmade slope failures, due to wrong or poor geotechnical engineering practices, are more common too. These are due to steep slopes or vertical cuts or near vertical cuts excavated. One example is shown in Figure 8.

Figure 8 Failure of a steep shedi soil slope adjacent to a building during monsoon season, aided by poor drainage.

There are problems of erosion of shedi soils in slopes, since the shedi soils are highly erosive by nature. Due to this highly erosive nature, in excavated slopes, especially for highway or railway purposes, caving-in of the shedi layers are a common sight, with the top lateritic crust being intact (Figure 7). Slopes with shedi soils, which are generally stable during dry seasons, fail during rainy seasons (Figure 8). Figure 9 shows a steep shedi soil slope excavated for a canal in the study area. It can be seen therein that the canal lining has failed due to erosion of the shedi soil behind it. It was recommended to provide a box culvert in such sections.

Figure 9 Damages caused to canal lining due to erosion of the lithomargic clay behind it

4. STABILIZATION OF LITHOMARGIC CLAYS (SHEDI SOILS)

A number of soil stabilization techniques are available in geotechnical engineering practice. In this study a few of the possible soil stabilization techniques were attempted to improve the behaviour of shedi soil and some of the results are reported in the following paragraphs.

4.1 Stabilizing with a Cementitious Stabilizer for Pavement Subgrades

4.1.1 Unconfined Compressive Strength Tests on Treated Soils under Unsoaked and Soaked Condition

One of the options while dealing with poor soils in pavement subgrades is stabilization. A proprietary cementitious stabilizer (Road Building International Grade 81 – RBI Grade 81) was used to improve the engineering properties of the shedi soil in pavements [Sample No. 20]. CBR of stabilized shedi soil under soaked and unsoaked conditions, under heavy compaction, have been studied by Sharath (2010). For unsoaked condition, the strength is found to increase with increase in curing period and also with increase in percentage of stabilizer (Tables 5 and 6). On soaking, there is a drastic reduction in strength of the stabilized soil. In soaked condition, there aren't any appreciable increase in strength with curing period, and also with increase in the percentage of the stabilizer

Table 5 Unconfined Compressive Strength Test Results on Treated Shedi Soil (Sample No.20) at MDD of Heavy Compaction [with the Proprietary Cementitious Stabilizer] for use in Pavements under Unsoaked Condition

Curing Period (Hrs/ Days)	UCC STRENGTH* IN MPa with 2% stabilizer	4% stabilizer	6% stabilizer
4 Hrs	0.950	1.090	1.350
1 day	1.050	1.700	1.870
3 days	1.220	1.850	2.020
7 days	1.550	2.050	2.360
28 days	1.820	2.580	3.100

* untreated shedi soil has UCS = 0.860 MPa

Table 6 Unconfined Compressive Strength Test Results on Treated Shedi Soil (Sample No.20) at MDD of Heavy Compaction [with the Proprietary Cementitious Stabilizer] for use in Pavements under Soaked Condition

Curing Period (Hrs/ Days)	UCC STRENGTH* IN MPa with 2% stabilizer	4% stabilizer	6% stabilizer
4 Hrs^s	0.023	0.028	0.033
1 day^s	0.040	0.079	0.091
3 days^s	0.050	0.093	0.120
7 days$^\#$	0.013	0.023	0.031
28 days$^\#$	0.060	0.105	0.190

* untreated (unsoaked) shedi soil has UCS = 0.860 MPa

^{s} cured for 4 hrs or 1 day or 3 days and soaked for 2 hours

\# cured for 7 days or 28 days and soaked for 1 day

4.1.2 CBR Test Results for Untreated and Treated Soils under Heavy Compaction (Sample No.20)

CBR tests on untreated shedi soil, after 7 days of moist curing and 4 days of soaking gave CBR value of 1.5%. These CBR values increased to 3.3%, 8.6% and 10.2%, after 7 days of moist curing and 4 days of soaking, with 2%, 4% and 6% stabilizer (Sharath, 2010).

4.1.3 Fatigue Life Test Results for Untreated and Treated Soils under Heavy Compaction (Sample No.20)

For design of semi-rigid pavements, fatigue life tests are more appropriate and are conducted to determine the response of treated soils for repeated loading conditions. The types of specimens tested for fatigue capacity of the stabilized soils, in this study, are similar to the UCC test specimens. Cylindrical specimens of height to diameter ratio of 2:1 are used, corresponding to heavy compaction characteristics of the soils. The repeated load testing machine used in this study is shown in Figure 10. It is a dynamic diametrical tensile test and the load is applied to the specimen in a positive sinusoidal pattern. The dynamic loading is applied using the hydraulic loading system present in the machine, and is transferred to the specimen through a movable shaft. A cooling system is attached to control temperature of the machine and pressure can be adjusted to balance between input and output loads. The specimen is fixed in between two steel strips present at the top and bottom of the testing setup. The position of the specimen is adjusted in such a way that it is exactly below the loading shaft and to apply the load along its diametrical plane. The specimen is connected with two vertical and two horizontal LVDTs, which measure the deflections. The machine is capable of applying load with frequency from 1 to 10 Hz and rest period 0 to 0.9 seconds. The machine is attached with a PC and can be controlled using a software 'fatigue 4.0', which is also used to provide various input values.

Figure 10 Repeated load testing machine

The repeated loading tests, in this study, are conducted at a frequency of 1 Hz. In fatigue tests, most of the pavement materials are tested at frequency of 1 Hz, which is the standard procedure (ASTM D 7369-11; Kallas and Puzinauskas, 1972). Tests are conducted after 7 days and 28 days of curing. Loads corresponding to one-third and half UCC strength for 2% stabilizer are applied for all the three sets of samples with different percentage of stabilizer. The results of the tests are tabulated in Table 7 (Sharath, 2010). Therein it can be seen that stabilization is effective in improving the fatigue or endurance life of soil samples. Treated specimens experience a large number of loading cycles before failure whereas their untreated counterparts fail within a few numbers of loading cycles.

Table 7 Fatigue Life Test Results on Untreated and Treated Shedi Soil (Sample No.20) at MDD of Heavy Compaction [with the Proprietary Cementitious Stabilizer] for use in Pavements

Curing Period in days & Load	FATIGUE LIFE* (number of cycles)		
	2% stabilizer	4% stabilizer	6% stabilizer
7 days -58.6 Kg	1732	3093	3961
7 days -87.8 Kg	19	173	317
28 days -68.8 Kg	2783	4317	5842
28 days -103.2 Kg	27	487	598

* untreated shedi soil failed between 2 to 4 cycles at all above loads after 7 or 28 days of curing.

4.1.4 Chemical Composition of Unblended and Blended Lithomargic Clays

Addition of stabilizer results in the formation of various chemicals which binds the soil particles together creating a crystalline matrix. This formation is evident from the chemical analysis of the treated and untreated soils, results of which are tabulated in Table 8. The analysis clearly shows an increase in percentages of calcium oxide, alumina and sulphates which are important byproducts, arising during stabilization and coefficient of permeability values. The main aim is to see if it meets the requirements of specific engineering projects, especially for its use as a pavement material. Coir materials are biodegradable and their uses in various geotechnical engineering applications are ecologically safe (Ravi Shankar et al. 2012a).

Table 8 Chemical Composition of Treated and Untreated Soils [Treated with the Proprietary Cementitious Stabilizer] (Sample No.20)

	1^C	2^C	3^C	4^C	5^C	6^C	7^C
Untreated	1.36	77.53	0.00	2.34	2.74	1.19	14.84
Treated	2.24	75.50	0.75	3.10	5.21	1.00	12.20

1^C is Ca(as CaO); 2^c is Si (as SiO_2); 3^c is S (as SO_4); 4^c is Al (as Al_2O_3); 5^c is Fe (as Fe_2O_3); 6^c is Mg (as MgO); 7^c is all others

4.2 Stabilizing Lithomargic Clay with Sand and Coir

The potential use of natural fibres in geotechnical engineering are for temporary applications in stabilization, for roads, embankments and railways, as well as for mulching in agricultural applications. In this study, investigations are conducted by blending the lithomargic clay with sand and coir to study the improvement in properties of the blended lithomargic clay such as UCC strength, CBR values.

4.2.1 Engineering Properties of Coir

The tensile strength of coir used, in its natural dry state, is 140 N/mm^2, and after 30 days of immersion in 10% normal sodium hydroxide solution its strength is 133 N/mm^2. The mean initial tangent modulus is around 5000 N/mm^2. It is the outer skin of the coir that effectively transmits the applied load. Therefore, it is the annular area of the outer skin alone and not the entire cross sectional area of the fibre that should be taken into account for the calculation of stress. The geometry of the fibre is not uniform throughout its length. Changes in the volume of the coir is found to be 5% as a result of wetting and drying tests, and water absorption of fibre body is found to be 59.5%. Coir has a number of small pits on its body and a comparatively large lumen (central cavity) at the centre. Most of the water taken in during the wet phase of the test fills up in this central cavity of the fibre body and hence there is a significant increase in the weight of the fibre when subject to the wetting and drying test with no appreciable change in volume. This behaviour is mainly due to the soft and porous nature of the fibre structure. The insignificant change in the fibre volume during the wetting and drying cyclic process has confirmed it as a dimensionally stable material. Coir is reported to be susceptible to the action of alkaline environment. Treatment of coir with either sodium hydroxide or calcium hydroxide causes a decrease in strength, as reported above. It has been reported that after a period of six months immersion in alkaline solution, the fibre strength will be reduced to almost nil with the change in material property from ductile to brittle.

The main chemical constituents of pure coir are cellulose (43.5%), lignin (46%) and hemi cellulose (0.2 - 0.3%). The mechanical properties are almost same as the synthetic fibres and they depend on the variety of the coir and the locality in which they are grown. The diameter/width of single fibre used in this study is about 16 microns and single fibre length used in the stabilization of shedi soil is 150 to 200 mm (Coir Board). The fibres are hygroscopic and its moisture absorption is 10-12% at 65% humidity and 22-25% at 95% humidity.

4.2.2 Laboratory Test Results on Coir and Sand Stabilized Soil

Based on the laboratory permeability tests conducted on blended soil, it was found that coefficient of permeability increases as the percentage of sand increases and decreases as the percentage of coir increases. The CBR both in soaked and unsoaked conditions increases as the percentage of sand increases from 0 to 40% and coir from 0 to 0.5% by weight of dry soil. UCC strength is found to increase with increase in percentage of coir, but in case of sand it increases only upto a certain percentage. It was concluded that the coir and sand modified soil can be used for highway embankments and for pavement subgrades.

The results of CBR and UCC test results on (i) unstabilized soil (ii) soil stabilized with 0.5% coir (iii) soil stabilized with 40% sand and (iv) soil stabilized with 0.5% coir and 40% sand; are shown tabulated in Tables 9 and 10 respectively. In Table 9, the effect of discrete coir fibres in improving CBR is very clear. Effect of fibres on soil plus sand is more than on soil alone. However with UCC test results, it can be seen that the effect of fibres on soil alone and that on soil blended with sand are nearly the same. Soaked CBR values of the coir and sand modified soil are substantially improved to values almost the same as the lithomargic clay alone under unsoaked condition.

4.2.3 Coir Mat (Geocoir) Reinforced Soil Subgrade

The effectiveness of coir geotextile reinforced soil subgrades in case of low volume rural roads is also being studied (Ravi Shankar et al. 2012b). CBR tests are conducted on soil and blended soil (blended with 50% sand). Plate load tests are conducted on soil, soil blended with sand and coir geotextile mat reinforced soil in case of both unblended and soils, to study their load deformation behavior. From Table 11, it is seen that the inclusion of geotextile coir mat is somewhat more effective for light compaction than for heavy compaction for both unblended and blended soils. In case of heavy compaction, inclusion of coir mat is seen to be beneficial only for unblended soil. Blending has definitely shown a significant increase in CBR values especially for soaked condition. Thus, introduction of a suitably modified soil layer above the existing sub-grade reduces the pavement section thickness.

Table 9 CBR Test Results on Unstabilized and Stabilized soils [Stabilized with Coir and Sand] (Sample No.24) [Ravi Shankar et al. 2012a]

Sl. No.	Description	CBR Value for unsoaked condition	CBR Value for Soaked Condition
LIGHT COMPACTION			
1	Lithomargic clay/soil	12.0	3.0
2	Lithomargic soil stabilized with 0.5% Coir	20.0	5.0
3	Lithomargic soil stabilized with 40% sand	21.0	5.0
4	Lithomargic soil stabilized with 40% sand and 0.5% coir	27.0	12.0
HEAVY COMPACTION			
5	Lithomargic clay/soil	16.0	4.0
6	Lithomargic soil stabilized with 0.5% Coir	25.0	7.0
7	Lithomargic soil stabilized with 40% sand	25.0	11.0
8	Lithomargic soil stabilized with 40% sand and 0.5% coir	33.0	16.0

Table 10 UCC Test Results on Unstabilized and Stabilized soils [Stabilized with Coir and Sand] (Sample No.24) [Ravi Shankar et al. 2012a]

Sl. No.	Description	q_u in MPa at Std. Proctor Density	q_u in MPa at Modified Proctor Density
1	Lithomargic clay/soil	0.27	0.38
2	Lithomargic soil stabilized with 0.5% coir	0.55	0.78
3	Lithomargic soil Stabilized with 40% sand	0.34	0.48
4	Lithomargic soil stabilized with 40% sand and 0.5% coir	0.54	0.77

Table 11 Results of CBR and Plate Load Tests on Coir Mat Reinforced Subgrade (Sample No.24) [Ravi Shankar et al. 2012b]

Sl. No.	Soil Description	CBR value (%)		Modulus of Subgrade Reaction (K) (N/mm^3)
		Soaked	Unsoaked	Unsoaked
LIGHT COMPACTION				
1	Original Soil	3.0	12.0	3.8
2	50% soil + 50% sand	5.0	22.0	2.6
3	Soil with Coir Mat	-	-	8.3
4	50% soil + 50% sand Coir Mat	-	-	4.4
HEAVY COMPACTION				
1	Original Soil	4.0	16.0	11.3
2	50% soil + 50% sand	11.0	27.0	11.8
3	Soil with Coir Mat	-	-	15.6
4	50% soil + 50% sand Coir Mat	-	-	10.1

5. GROUND IMPROVEMENT OF SHEDI GROUNDS

5.1 Ground Improvement for Footings on Shedi Ground using Geogrids

5.1.1 Granular Bed (GB) overlying Poor Shedi Ground

Placing a dense granular bed over weak ground is the simplest ground improvement technique for improving bearing capacity of weak grounds. Granular bed have been extensively used to support ground level storage tanks over weak, soft and compressible shedi grounds in this study area.

5.1.2 Geogrid Reinforced Granular Bed (RGB) overlying Poor Shedi Ground

The bearing capacity of granular bed over weak grounds can be further improved by providing a geosynthetic layer at the interface of the weak soil and the fill. Geosynthetic reinforcement can also be provided in layers in the overlying granular bed itself. The use of geosynthetic reinforced granular bed over soft soil effectively reduces settlement and increases the bearing capacity of soft soil. Shivashankar et al. (1993) proposed a punching shear failure mechanism in which both the footing and the portion of the reinforced granular bed directly beneath the footing are envisaged to act in unison to punch through the soft soil underneath. The improvement in bearing capacity of a reinforced granular bed is attributed to three effects namely Shear layer effect, Confinement effect and Surcharge effect (Sivakumar Babu, 2006).

5.1.3 Prestressed Geogrid Reinforced Granular Bed (PRGB) overlying Poor Shedi Ground

Geosynthetics are found to demonstrate their beneficial effects only after considerable settlements, since the strains occurring during initial settlements are insufficient to mobilize significant tensile load in the geosynthetic. This is not a desirable feature for foundations of certain types of structures, since their permissible values of settlement are low. Thus a need for a technique which will allow the geosynthetic to increase the load bearing capacity of ground without the occurrence of large settlements was felt (Jayamohan and Shivashankar, 2012; Shivashankar and Jayaraj, 2014). The settlements of a reinforced granular bed can be considerably reduced by prestressing the geosynthetic reinforcement.

The effects of prestressing the reinforcement in the RGB on the load-bearing capacity and settlement response of a prestressed reinforced granular bed overlying soft soil are also studied through laboratory scale model tests and numerical modeling using FEA software PLAXIS. The parameters studied are the effects of thickness of granular bed, magnitude of prestress, direction of prestress and strength of the weak soil. Addition of prestress to reinforcement was found to significantly improve the bearing capacity and settlement behaviour of the shedi soil ground. Prestressing with 2% of the tensile strength of the geosynthetic was found to significantly increase the bearing capacity and reduce the settlements as seen in Figure 11, both from laboratory study as well as FEA. Uniaxial prestressing was found to be better than biaxial prestressing (Jayamohan and Shivashankar, 2012; Shivashankar and Jayaraj, 2014)

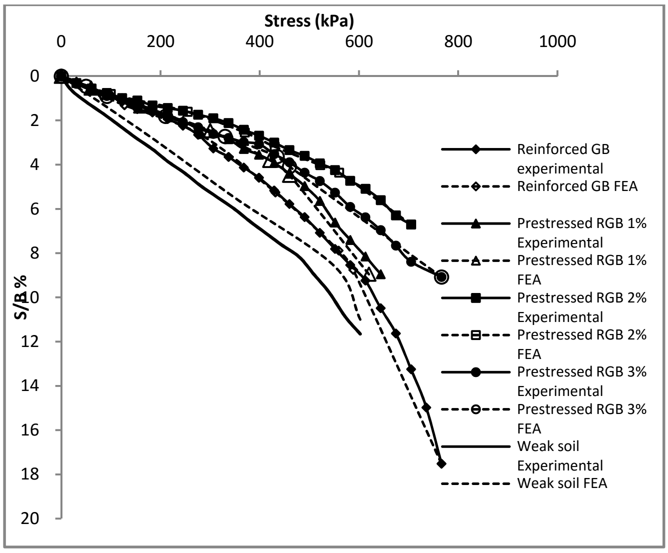

Figure 11 Stress versus normalized settlement curves for granular beds (RGB, PRGB) of thickness B overlying (moist) weak shedi ground, from laboratory scale plate load test results

5.2 Geogrid Reinforced Shedi Soil Subgrade (Sample No.25)

Plate load tests were conducted at soaked and unsoaked conditions for unreinforced and reinforced (with geogrid) subgrade. The properties of the shedi soil (Sample No.25) used are shown in Tables 1 to 4. Properties of the HDPE geogrid used were ascertained as per ASTM standards. Tensile strength in machine direction was 7.84 kN/m and in cross-machine direction was 6.34 kN/m. Elongation at maximum load was 42.4% in machine direction and 34.0% in cross-machine direction. Details of the plate load tests are given in Ravi Shankar and Suresha, 2006. Summary of the test results are shown tabulated in Table 12. It was concluded that the reduction in elastic modulus and modulus of subgrade reaction for soaked unreinforced subgrade were about 70% and 60% respectively, when compared with that of the unsoaked unreinforced condition. The inclusion of geogrid reinforcement in the soaked subgrade, elastic modulus improved by about 33% and modulus of subgrade reaction by about 43% as compared to soaked and unreinforced case.

5.3 Ground Improvement by Inclusions in the Shedi Soil Ground - Reinforced Stone Columns (RSC)

Stone columns or granular piles is a ground improvement technique wherein the improvement is due to a combination of densification during installation, reinforcement and drainage. The advantages of stone columns are increase in bearing capacity (3 to 4 times),

Table 12 Modulii of Elasticity and Modulii of Subgrade Reaction under Light Compaction for Unreinforced and Geogrid Reinforced Subgrades [Ravi Shankar and Suresha, 2006]

Sl. No.	Particulars	E (MPa)	K (N/mm^3)
UNSOAKED CONDITION			
1	Unreinforced Subgrade	10.584	142.24 X 10^3
2	Geogrid reinforced subgrade, spacing = 1.5 times plate diameter	15.024	242.72 X 10^3
3	Geogrid reinforced subgrade, spacing = 0.75 times plate diameter	16.180	343.20 X 10^3
4	Geogrid reinforced subgrade, spacing = 0.5 times plate diameter	17.335	418.56 X 10^3
SOAKED CONDITION			
5	Unreinforced Subgrade	3.178	58.59 X 10^3
6	Geogrid reinforced subgrade, spacing = 0.5 times plate diameter	4.237	83.70 X 10^3
7	Reinforced subgrade - with two layers of geogrid, at 0.5a and 0.75a from top, a = plate diameter	NA	125.56 X 10^3

reduction in settlements to the extent of 50%, accelerated consolidation process, reduction in the liquefaction potential of sandy deposits during earthquakes. Moreover, it is economical and it easy to construct, and speedy construction is possible. It is most suitable for tank foundations, earth embankments, low rise buildings and other flexible structures. Stone columns derive their strength from bulging and the lateral confinement provided by the surrounding soil. However in very soft soils, the stone columns may not derive significant load carrying capacity due to low lateral confinement and also there will be excessive bulging and excessive settlements. There is also possibility of squeezing of soft soil into the stones. Encasing the stone column with a geosynthetic fabric (Encased Stone Columns or ESC) has been studied by many researchers (Van Impe, 1989; Katti et al. 1993; Bauer and Al-Joulani 1994; Raithel et al. 2002; Ayadat and Hanna, 2005; Murugesan and Rajagopal, 2006, 2007, 2008, 2010; Malarvizhi and Ilamparuthi, 2007). Load carrying capacity is increased and bulging decreased with the use of ESC. Encasing is required for top 2 to 3 times the diameter to significantly enhance the performance of the stone columns. Limitation of ESC is that it will not allow the column to dilate and accordingly to increase the in-situ stresses. Compaction of stone column is kept to as minimum as possible to avoid damage to the geotextile during installation, as a result of which large settlements might occur (Gneil and Bouazza, 2008). Construction of ESC is somewhat difficult in site. Therefore, stone columns reinforced with vertical reinforcements (nails) along the circumference were studied in weak shedi grounds (Shivashankar et al. 2010, 2011; Sitaram Nayak et al 2011, Dheerendra Babu et al. 2010, Dheerendra Babu, 2011). Effects of depth of nails, number of nails (n), area ratio (A_r), diameter of stone columns (D), end condition (floating piles or end bearing piles) were studied in detail from laboratory scale model tests. Some results are presented herein. Figure 12 shows a typical test arrangement for 90 mm stone column, with vertical circumferential nails surrounding the stone columns in shedi ground, for column area loading and entire area loading.

In the field, the entire area of the stone column area treated ground is subjected to loading from the superstructure. The same was simulated in the laboratory by loading the whole area of the unit cell. These tests are used to study the improvement in load carrying capacity and reduction in the settlement of the treated ground. Tests in which only the column area was loaded were used to find the improvement in limiting axial capacity of the reinforced stone column (RSC) over the plane stone column (PSC).

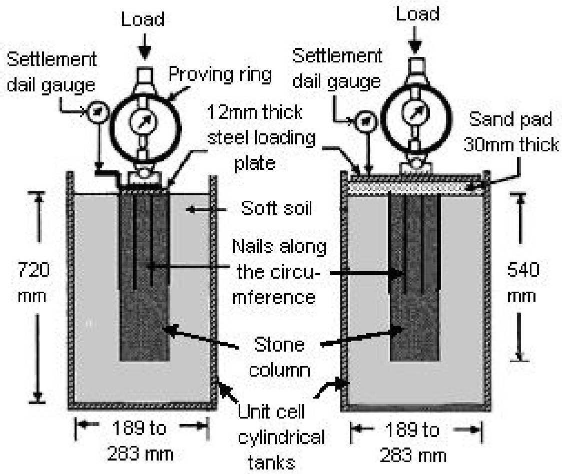

Figure 12 Typical test arrangement for 90 mm column, with vertical circumferential nails surrounding the stone columns in shedi ground, for (a) column area loading and (b) entire area loading

Figure 13 shows the effect of depth of nails (h) for the case of entire area loading, with reinforced stone column (RSC) using 4mm diameter (d) nails and 8 number of nails (n). Therein, it can be seen that there is not much benefit beyond h = 3D. Figure 14 shows the effect of number of nails (n) for entire area loading, with reinforced stone column (RSC) using 4mm diameter nails and depth of nails (h) being 3D. Therein it can be seen that the improvement increases with the increase in the number of nails. Figures 15 and 16, show the effect of diameter of stone columns (D). Therein it can be seen that the smaller diameter stone columns perform better than the large diameter stone columns for plane stone columns (PSC) and reinforced stone columns (RSC) for an area ratio (A_r) of 15%. The effect of nails on bulging behavior is shown in Figure 17. It is seen that the bulging is significantly reduced (more than 60%) in the case of reinforced stone columns. Figure 17 shows that as the area ratio increases for both unreinforced and reinforced stone columns, the lateral bulging decreases. Bulging typically happens in top 3D. It was observed that for an area ratio of 15%, the bulging increases with decrease in the diameter of the stone columns. For smaller diameter stone columns the reduction in bulging was as much as 80% for reinforced stone column as compared to plane stone columns. It was concluded that the performance of stone columns installed in soft soils can be significantly enhanced by reinforcing the stone columns with circumferential nails. The improvement increases with the number of nails and diameter of nails. The depth of embedment of nails up to 3D depth is sufficient to significantly enhance the performance of stone columns. The nailing method is an effective alternative and a practically feasible method to enhance performance of stone columns. This nailing can be done post construction of stone columns.

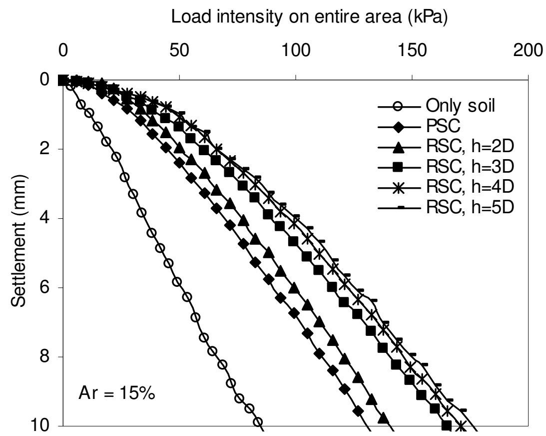

Figure 13 Effect of depth of nails for entire area loading, for RSC with d = 4 mm and n = 8 (Reinforced stone column); d is diameter of nails, n is number of nails, h is depth of nails, A_r is Area Ratio

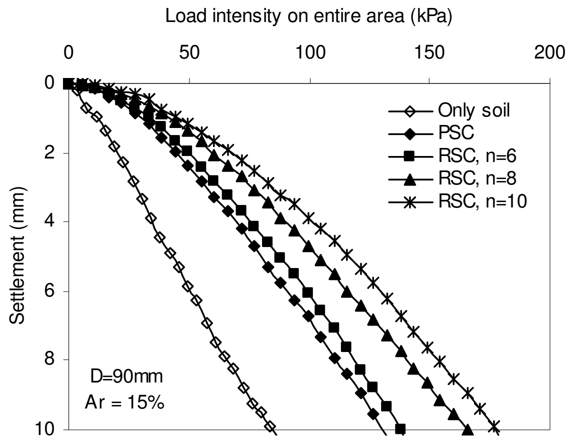

Figure 14 Effect of number of nails for entire area loading, for RSC with d = 4mm, h = 3D (Reinforced stone column) and A_r is Area Ratio

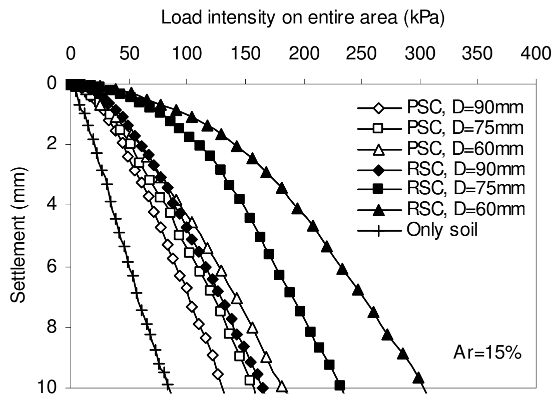

Figure 15 Effect of diameter of stone columns, for entire area loading, for unreinforced and reinforced stone columns. A_r is the Area ratio [d = 4 mm, n = 8 and h = 3D]

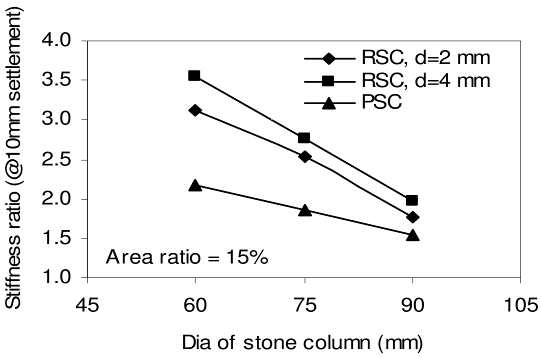

Figure16 Effect of diameter of stone column and diameter of nail for both unreinforced and reinforced stone columns in Shedi Ground

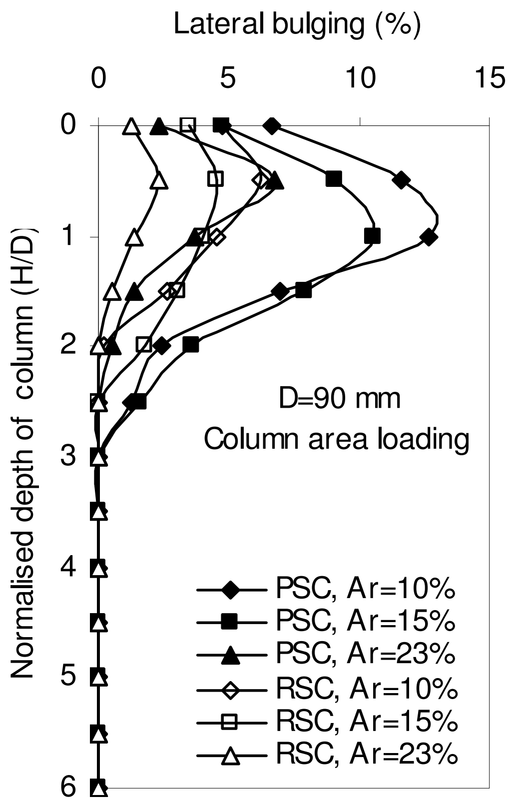

Figure 17 Effect of reinforcement and area ratio (A_r) on lateral bulging of stone column

6. EROSION CONTROL AND SLOPE PROTECTION

Erosion studies (internal erosion studies) through laboratory experiments indicated that though lateritic lithomarge soils, due to a higher percentage of fine fractions, erode at higher head, the wash out will be sudden, unlike the lateritic soils with a higher fraction of coarse fractions (Rajeshwari, 2011). Embankment slopes must be engineered. Good drainage must be provided to prevent internal erosion and surface of slope must be properly protected against surface erosion. Surface erosion could also gradually lead to slope instability (Figures. 7, 8 and 9). Case studies of slope failures and landslides in this area have been reported earlier (Setty et al. 1999, Bhat et al. 2008 etc.). These days the common practice is to turf the slopes with vetiver grass to protect slopes against surface erosion. Figure 18 shows a photo of an excavated slope along the Konkan railway track protected with vetiver grass for erosion control. In many places, in the present study area, slopes are protected by gabion toe walls (Figure 19).

Figure 18 A shedi soil slope along the Konkan railway track Planted with vetiver saplings for erosion control

Figure 19 Another section of Konkan railway, where the slope is protected by gabion wall

6.1 Effect of Vegetation on Slope Stability of Shedi Soil Slopes

Vetiver, is a very fast growing grass and until very recently a relatively unknown plant in this area. It possesses some unique features of both grasses and trees by having profusely grown, deep penetrating root system. The roots of vetiver grass can offer both erosion prevention and control of shallow movement of surficial earth mass. Vetiver grass roots are very strong with an average tensile strength of 75 Mpa or about one-sixth of ultimate strength of mild steel. In addition to its unique morphological characteristics, vetiver is also highly tolerant to adverse growing conditions such as extreme soil pH, temperatures and heavy metal toxicities (Paul, 1999). The massive root system also increases the shear strength of soil, thereby enhancing slope stability appreciably. Shivashankar et al. (2014b) have studied the effect of vegetation, including turfing such as growing vetiver grass on slopes and effect of trees on slopes, on stability of slopes. They conclude that in most cases, the effect of vegetation is beneficial in controlling surface erosion and improving the stability of slopes.

7. CONCLUSIONS

All soils are good in dry condition. Lithomargic clays, locally called as shedi soils, are products of laterization. They occur sandwiched between the top hard lateritic crust and the parent rock, which is granitic gneiss, beneath. Lateritic lithomarges comprising of 75 to 100% of the lithomargic clay are quite often exposed when the top laterite crust is removed. Lowlying areas are also filled up with the lateritic lithomarges for construction purposes. Lateritic lithomarges generally classify as sandy silt or silty sand, with little or no clay size particles. The particle size of sands present in the lateritic lithomarges corresponds to fine sand. Lateritic lithomarges and lithomargic clays are very sensitive to moisture variations, especially when there is no confinement. They behave like dispersive soils. Problems of dealing with shedi soils for highway engineers and foundation engineers, are loss of shear strength on wetting and removal of confinement, erosion problems, landslides, slope stability problems etc.

The properties of shedi soil subgrades can be substantially improved by blending it with suitable stabilizers. A number of stabilizers are available, including natural materials like coir.

Modification of ground by inclusions is a very effective method of ground improvement in improving the bearing capacity and reducing the settlements of poor grounds. Inclusions can be by way of geogrid reinforcements laid horizontally in a granular bed overlying poor grounds or by way of stone columns. Stone columns consist essentially of replacing subsoil in weak grounds with compacted stones or stronger granular material in pre-bored vertical holes, to form columns (stone columns) or piles (granular piles) within the soil.

Slopes are to be engineered, and most importantly provided with proper drainage facilities and erosion control measures at site, for good performance of civil engineering systems. Vetiver is a very effective solution in tackling erosion control problems.

8. ACKNOWLEDGEMENTS

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9. REFERENCES

ASTM D7369-11 "Standard test method for determining the resilient modulus of bituminous mixtures by indirect tension test", American Society of Testing and Materials (ASTM)

Ayadat, T. and Hanna, A.M. (2005) "Encapsulated stone columns as a soil improvement technique for collapsible soil". Ground Improvement, 9(4), pp137-147.

Bauer, G.E., and Al-Joulani, N. (1994) "Strengthening of marginal soils by confined stone columns", Proceedings of 7th international cold regions engineering—A global perspective, Edmonton, Alberta specialty conference, University of Alberta, Edmonton, Alberta, pp 97–117.

Bhat, A. K ., Shivashankar, R., and Yaji, R. K., (2008) "Case study of landslide in NH13 at Kethikal near Mangalore, India", Proceedings of the Sixth Intl. Conf. on Case Histories in Geotech. Engg., Arlington, VA, USA: 2.69-2.71.

Buchanan-Hamilton, Francis, (1809) extracted from http://en.wikipedia.org/wiki/Laterite

Dheerendra Babu, M.R., Shivashankar, R.., and Sitaram Nayak, (2010) "Load settlement behavior of stone columns with circumferential nails", Proceedings of the Indian Geotechnical Conference, Mumbai, Theme T8, Abstract No.188.

Dheerendra Babu, M.R. (2011) Studies on Behaviour of Plain and Reinforced Stone Columns in Soft and Layered soils, Ph.D. Thesis, National Institute of Technology Karnataka-Surathkal, Mangalore, India, p162

Gidigasu, M. D. (1972) "Mode of formation and geotechnical characteristics of laterite materials of Ghana in relation to soil forming factors". Engineering Geology Journal. Volume 6, issue 2, August 1972, pp.79-150.

Gidigasu, M. D. (1976) Laterite Soil Engineering - Pedogenesis and Engineering Principles. Developments in Geotechnical Engineering. Elsevier Scientific Publishing Company, p554

Gniel, J., and Bouazza, A., (2008) "Improvement of soft soils using geogrid encased stone columns". Journal of Geotextiles and Geomembranes, 27(3), pp167-175.

Jayamohan, J. and Shivashankar, R., (2012) "Some studies on prestressed reinforced granular beds overlying weak soil". ISRN Journal of Civil Engineering, Article id 436327.2012, Hindawi Publishers.

Jayamohan, J. (2014) Behaviour of Prestressed Reinforced Granular Bed overlying Soft Soil. Ph.D. Thesis, National Institute of Technology Karnataka - Surathkal, Mangalore, India, p162

Kallas, B. F. and Puzinauskas, V.P., (1972) "Flexural fatigue tests on asphalt paving mixtures - fatigue of compacted bituminous aggregate mixtures" American Society of Testing and Materials (ASTM), STP 508.

Katti, R.K., Katti, A.R. and Nayak, S. (1993) "Monograph to analysis of stone columns with and without geosynthetic encasement", CBIP Publication, New Delhi

Khanna, S.K. and Justo, C.E.G., (1991) Highway Engineering. 7th Edition, Nem Chand and Bros., Roorkee, India, p862.

Krishna Murthy, G. (1999) Ground Improvement Techniques for Shedi Ground. M.Tech Thesis, National Institute of Technology Karnataka - Surathkal, Mangalore, India, p64.

Malarvizhi, S.N. and Ilamparuthi, K. (2007) "Comparative study on the behavior of encased stone column and conventional stone column". Soil Found 47(5):873–886.

Morin, W.J. and Todor, Peter C. (1975) "Laterite and lateritic soils and other problem soils of the tropics". Volumes I and II Instruction Manuals. An engineering evaluation and highway design study for USAID AID/csd 3682.

Murugesan, S. and Rajagopal, K. (2006) "Geosynthetic encased stone columns: numerical evaluation". Journal of Geotextiles and Geomembranes 24(6):349–358.

Murugesan, S. and Rajagopal, K. (2007) "Model tests on geosynthetic encased stone columns". Geosynthetics Intl. Journal 24(6):346–354.

Murugesan, S. and Rajagopal, K. (2008) "Shear load tests on stone columns with and without geosynthetic encasement". Geotechnical testing journal, ASTM, 32(1).

Murugesan, S. and Rajagopal, K. (2010) "Studies on the behaviour of single and group of geosynthetic encased stone columns". ASCE, J Geotech Geoenviron Eng 136(1):129–139.

Paul N.V. Truong (1999) "Vetiver grass technology for land stabilization, erosion control in the Asia Pacific region", Proceedings of the First Asia-Pacific Conference on Ground and Water Bio-engineering, Manila, April 1999 http://www.vetiver.org/TVN_vgt.htm.

Raithel, M., Kempfert, H.G., and Kirchner, A. (2002) "Geotextile encased columns (GEC) for foundation of a dike on very soft soils", Proceedings of the 7th international conference on geosynthetics, Nice, France, pp. 1025–1028.

Rajeshwari, B. (2011) Experimental Studies on Erosion Potential of Shedi and Lateritic soils. M.Tech Thesis, National Institute of Technology Karnataka - Surathkal, Mangalore, India, p85

Rao, Ch.Nageshwar (2008) Studies on Strength and Stiffness Characteristics of Lateritic Soils for Pavement Subgrades. Ph.D. Thesis, National Institute of Technology Karnataka - Surathkal, Mangalore, India, p254

Ravi Shankar, A. U., and Suresha, S. N., (2006) "Strength behaviour of geogrid reinforced shedi soil subgrade and aggregate system". International Journal of Road Materials and Pavement Design, Lavoisier, Vol. 7 - No.3/2006, pp313-330.

Ravi Shankar, A. U., Chandrasekhar, A., and Prakash Bhat, H., (2012a) "Experimental investigation on lithomargic clay stabilized with sand and coir". Indian Highways Journal, February 2012, pp21-31.

Ravi Shankar, A.U., Shivashankar, R., and Sachit Pai, N., (2012b) "Load deformation behaviour of coir mat treated soil", Proceedings of the International Conference on Ground Improvement and Ground Control (ICGI 2012), Uni. of Wollongong, Australia, Published by Research and Publishing, Singapore, Vol 2, pp1571-1576.

Roy Chowdhury, M. K., Venkatesh, V., Anandalwar, M. A. and Paul, D. K., (1965) "Recent concepts on the origin of Indian laterite". Memoirs of Geological Survey of India, 31A(6), pp. 547-558.

Setty, K.R.N., Shivashankar, R., and Lakshman Rao Peshwe, (1999) Kethikal slope failure – A case study. Proc. Indian Geotech. Conf. Calcutta.

Sharath, P. (2010) Studies on Soils Treated with a Cementitious Stabilizer for use as Pavement Materials. M.Tech Thesis, National Institute of Technology Karnataka-Surathkal, Mangalore, India, p94

Shivashankar, R., Madhav, M.R., and Miura, N., (1993) “Reinforced Granular Beds Overlying Soft Clay”, Proceedings of the 11th Southeast Geotechnical Conference, Singapore, pp409-414.

Shivashankar, R., Dheerendra Babu, M.R., Sitaram Nayak, and Manjunath, R., (2010) "Stone columns with vertical circumferential nails: A laboratory model study". Journal of Geotechnical and Geological Engineering, Springer Publishers, Netherlands, Volume 28, No.5, pp695-706.

Shivashankar, R., Dheerendra Babu, M.R., Sitaram Nayak, and Rajath Kumar, V., (2011) "Experimental studies on behaviour of stone columns in layered soils". Journal of Geotechnical and Geological Engineering, Springer Publishers, Netherlands, Volume 29, No.5, pp749-757.

Shivashankar, R., and Jayaraj, J., (2014) "Effects of prestressing the reinforcement on the behaviour of reinforced granular beds overlying weak soil". Geotextiles and Geomembranes, Elsevier publishers Vol.42, Issue 1, pp69-75 http://dx.doi.org/10.1016/j.geotexmem.2013.08.008

Shivashankar, R., Ravi Shankar, A. U., and Jayamohan, J.,(2014a) "Studies on engineering properties of lithomargic clays of coastal Karnataka in India", Proceedings of the 9th International Symposium on Lowland Technology "Problems and remedial measures of lowland", organized by ISLT, September 29 to October 01, 2014 at Saga, Japan, pp93-100.

Shivashankar, R., Biji, C.T., Hridya, S. and Venkat Reddy, D., (2014b) "Effect of vegetation on shear strength of shedi soils and slope stability", Proceedings of the Indian Geotechnical Conference (IGC), JNTU, Kakinada, 18-20 December 2014, 10 pages, pg. 269 in volume of abstracts.

Sitaram Nayak, Ramaiah Shivashankar, and Madhalam Ramanaidu Dheerendra Babu, (2011) "Performance of stone columns with circumferential nails". Proceedings of the ICE – Ground Improvement, ISSN 1755-0750, E-ISSN 1755-0769, Volume 164, Issue 2, pp97-106.

SP36 Part 1 (1987) "Compendium of Indian standards on soil engineering - Part 1 Laboratory testing of soils for civil engineering purposes". Bureau of Indian Standards.

Sivakumar Babu, G. L. (2006) Introduction to Soil Reinforcement and Geosynthetics. Universities Press, Hyderabad, p200.

Van Impe, W.F. (1989) Soil improvement techniques and their evolution. Balkema, Rotterdam www.coirboard.gov.in